Chapter 1
Introduction

1-1. Purpose and Scope

This engineer manual (EM) provides guidance for analyzing the static stability of slopes of earth and rock-fill dams, slopes of other types of embankments, excavated slopes, and natural slopes in soil and soft rock. Methods for analysis of slope stability are described and are illustrated by examples in the appendixes. Criteria are presented for strength tests, analysis conditions, and factors of safety. The criteria in this EM are to be used with methods of stability analysis that satisfy all conditions of equilibrium. Methods that do not satisfy all conditions of equilibrium may involve significant inaccuracies and should be used only under the restricted conditions described herein. This manual is intended to guide design and construction engineers, rather than to specify rigid procedures to be followed in connection with a particular project.

1-2. Applicability

This EM is applicable to all USACE elements and field operating activities having responsibility for analyzing stability of slopes.

1-3. References

Appendix A contains a list of Government and non-Government references pertaining to this manual. Each reference is identified in the text by either the designated publication number or by author and date.

1-4. Notation and Glossary

Symbols used in this manual are listed and defined in Appendix B. The notation in this manual corresponds whenever possible to that recommended by the American Society of Civil Engineers.

1-5. Basic Design Considerations

a. General overview. Successful design requires consistency in the design process. What are considered to be appropriate values of factor of safety are inseparable from the procedures used to measure shear strengths and analyze stability. Where procedures for sampling, testing, or analysis are different from the procedures described in this manual, it is imperative to evaluate the effects of those differences.

b. Site characterization. The stability of dams and slopes must be evaluated utilizing pertinent geologic information and information regarding in situ engineering properties of soil and rock materials. The geologic information and site characteristics that should be considered include:

(1) Groundwater and seepage conditions.

(2) Lithology, stratigraphy, and geologic details disclosed by borings and geologic interpretations.

(3) Maximum past overburden at the site as deduced from geological evidence.

(4) Structure, including bedding, folding, and faulting.

(5) Alteration of materials by faulting.

(6) Joints and joint systems.

(7) Weathering.

(8) Cementation.

(9) Slickensides.

(10) Field evidence relating to slides, earthquake activity, movement along existing faults, and tension jointing.

c. Material characterization. In evaluating engineering properties of soil and rock materials for use in design, consideration must be given to: (1) possible variation in natural deposits or borrow materials, (2) natural water contents of the materials, (3) climatic conditions, (4) possible variations in rate and methods of fill placement, and (5) variations in placement water contents and compacted densities that must be expected with normal control of fill construction. Other factors that must be considered in selecting values of design parameters, which can be evaluated only through exercise of engineering judgment, include: (1) the effect of differential settlements where embankments are located on compressible foundations or in narrow, deep valleys, and (2) stress-strain compatibility of zones of different materials within an embankment, or of the embankment and its foundation. The stability analyses presented in this manual assume that design strengths can be mobilized simultaneously in all materials along assumed sliding surfaces.

d. Conventional analysis procedures (limit equilibrium). The conventional limit equilibrium methods of slope stability analysis used in geotechnical practice investigate the equilibrium of a soil mass tending to move downslope under the influence of gravity. A comparison is made between forces, moments, or stresses tending to cause instability of the mass, and those that resist instability. Two-dimensional (2-D) sections are analyzed and plane strain conditions are assumed. These methods assume that the shear strengths of the materials along the potential failure surface are governed by linear (Mohr-Coulomb) or nonlinear relationships between shear strength and the normal stress on the failure surface.

(1) A free body of the soil mass bounded below by an assumed or known surface of sliding (potential slip surface), and above by the surface of the slope, is considered in these analyses. The requirements for static equilibrium of the soil mass are used to compute a factor of safety with respect to shear strength. The factor of safety is defined as the ratio of the available shear resistance (the capacity) to that required for equilibrium (the demand). Limit equilibrium analyses assume the factor of safety is the same along the entire slip surface. A value of factor of safety greater than 1.0 indicates that capacity exceeds demand and that the slope will be stable with respect to sliding along the assumed particular slip surface analyzed. A value of factor of safety less than 1.0 indicates that the slope will be unstable.

(2) The most common methods for limit equilibrium analyses are methods of slices. In these methods, the soil mass above the assumed slip surface is divided into vertical slices for purposes of convenience in analysis. Several different methods of slices have been developed. These methods may result in different values of factor of safety because: (a) the various methods employ different assumptions to make the problem statically determinate, and (b) some of the methods do not satisfy all conditions of equilibrium. These issues are discussed in Appendix C.

e. Special analysis procedures (finite element, three-dimensional (3-D), and probabilistic methods).

(1) The finite element method can be used to compute stresses and displacements in earth structures. The method is particularly useful for soil-structure interaction problems, in which structural members interact with a soil mass. The stability of a slope cannot be determined directly from finite element analyses, but the

computed stresses in a slope can be used to compute a factor of safety. Use of the finite element method for stability problems is a complex and time-consuming process. Finite element analyses are discussed briefly in Appendix C.

(2) Three-dimensional limit equilibrium analysis methods consider the 3-D shapes of slip surfaces. These methods, like 2-D methods, require assumptions to achieve a statically determinate definition of the problem. Most do not satisfy all conditions of static equilibrium in three dimensions and lack general methodologies for locating the most critical 3-D slip surface. The errors associated with these limitations may be of the same magnitude as the 3-D effects that are being modeled. These methods may be useful for estimating potential 3-D effects for a particular slip surface. However, 3-D methods are not recommended for general use in design because of their limitations. The factors of safety presented in this manual are based on 2-D analyses. Three-dimensional analysis methods are not included within the scope of this manual.

(3) Probabilistic approaches to analysis and design of slopes consider the magnitudes of uncertainties regarding shear strengths and the other parameters involved in computing factors of safety. In the traditional (deterministic) approach to slope stability analysis and design, the shear strength, slope geometry, external loads, and pore water pressures are assigned specific unvarying values. Appendix D discusses shear strength value selection. The value of the calculated factor of safety depends on the judgments made in selecting the values of the various design parameters. In probabilistic methods, the possibility that values of shear strength and other parameters may vary is considered, providing a means of evaluating the degree of uncertainty associated with the computed factor of safety. Although probabilistic techniques are not required for slope analysis or design, these methods allow the designer to address issues beyond those that can be addressed by deterministic methods, and their use is encouraged. Probabilistic methods can be utilized to supplement conventional deterministic analyses with little additional effort. Engineering Technical Letter (ETL) 1110-2-556 (1999) describes techniques for probabilistic analyses and their application to slope stability studies.

f. Computer programs and design charts. Computer programs provide a means for detailed analysis of slope stability. Design charts provide a rapid method of analysis but usually require simplifying approximations for application to actual slope conditions. The choice to use computer programs or slope stability charts should be made based on the complexity of the conditions to be analyzed and the objective of the analysis. Even when computer programs are used for final analyses, charts are often useful for providing preliminary results quickly, and for providing an independent check on the results of the computer analyses. These issues are discussed in Appendix E.

g. Use and value of results. Slope stability analyses provide a means of comparing relative merits of trial cross sections during design and for evaluating the effects of changes in assumed embankment and foundation properties. The value of stability analyses depends on the validity of assumed conditions, and the value of the results is increased where they can be compared with analyses for similar structures where construction and operating experiences are known.

h. Strain softening and progressive failure. "Progressive failure" occurs under conditions where shearing resistance first increases and then decreases with increasing strain, and, as a result, the peak shear strengths of the materials at all points along a slip surface cannot be mobilized simultaneously. When progressive failure occurs, a critical assumption of limit equilibrium methods – that peak strength can be mobilized at all points along the shear surface -- is not valid. "Strain softening" is the term used to describe stress-strain response in which shear resistance falls from its peak value to a lower value with increasing shear strain. There are several fundamental causes and forms of strain softening behavior, including:

(1) Undrained strength loss caused by contraction-induced increase in pore water pressure. Liquefaction of cohesionless soils is an extreme example of undrained strength loss as the result of contraction-induced pore pressure, but cohesive soils are also subject to undrained strength loss from the same cause.

(2) Drained strength loss occurring as a result of dilatancy. As dense soil is sheared, it may expand, becoming less dense and therefore weaker.

(3) Under either drained or undrained conditions, platy clay particles may be reoriented by shear deformation into a parallel arrangement termed "slickensides," with greatly reduced shear resistance. If materials are subject to strain softening, it cannot be assumed that a factor of safety greater than one based on peak shear strength implies stability, because deformations can cause local loss of strength, requiring mobilization of additional strength at other points along the slip surface. This, in turn, can cause additional movement, leading to further strain softening. Thus, a slope in strain softening materials is at risk of progressive failure if the peak strength is mobilized anywhere along the failure surface. Possible remedies are to design so that the factor of safety is higher, or to use shear strengths that are less than peak strengths. In certain soils, it may even be necessary to use residual shear strengths.

i. Strain incompatibility. When an embankment and its foundation consist of dissimilar materials, it may not be possible to mobilize peak strengths simultaneously along the entire length of the slip surface. Where stiff embankments overly soft clay foundations, or where the foundation of an embankment consists of brittle clays, clay shales, or marine clays that have stress-strain characteristics different from those of the embankment, progressive failure may occur as a result of strain incompatibility.

j. Loss of strength resulting from tension cracks. Progressive failure may start when tension cracks develop as a result of differential settlements or shrinkage. The maximum depth of cracking can be estimated from Appendix C, Equation C-36. Shear resistance along tension cracks should be ignored, and in most cases it should be assumed that the crack will fill with water during rainfall.

k. Problem shales. Shales can be divided into two broad groups. Clay shales (compaction shales) lack significant strength from cementation. Cemented shales have substantial strength because of calcareous, siliceous, other types of chemical bonds, or heat, and pressure. Clay shales usually slake rapidly into unbonded clay when subjected to a few cycles of wetting and drying, whereas cemented shales are either unaffected by wetting and drying, or are reduced to sand-size aggregates of clay particles by wetting and drying. All types of shales may present foundation problems where they contain joints, shear bands, slickensides, faults, seams filled with soft material, or weak layers. Where such defects exist, they control the strength of the mass. Prediction of the field behavior of clay shales should not be based solely on results of conventional laboratory tests, since they may be misleading, but on detailed geologic investigations and/or large-scale field tests. Potential problem shales can be recognized by: (1) observation of landslides or faults through aerial or ground reconnaissance, (2) observation of soft zones, shear bands, or slickensides in recovered core or exploration trenches, and (3) clay mineralogical studies to detect the presence of bentonite layers.

1-6. Stability Analysis and Design Procedure

The process of evaluating slope stability involves the following chain of events:

a. Explore and sample foundation and borrow sources. EM 1110-1-1804 provides methods and procedures that address these issues.

b. Characterize the soil strength (see Appendix D). This usually involves testing representative samples as described in EM 1110-2-1906. The selection of representative samples for testing requires much care.

c. Establish the 2-D idealization of the cross section, including the surface geometry and the subsurface boundaries between the various materials.

d. Establish the seepage and groundwater conditions in the cross section as measured or as predicted for the design load conditions. EM 1110-2-1901 describes methods to establishing seepage conditions through analysis and field measurements.

e. Select loading conditions for analysis (see Chapter 2).

f. Select trial slip surfaces and compute factors of safety using Spencer's method. In some cases it may be adequate to compute factors of safety using the Simplified Bishop Method or the force equilibrium method (including the Modified Swedish Method) with a constant side force (Appendix C). Appendix F provides example problems and calculations for the simplified Bishop and Modified Swedish Procedures.

g. Repeat step f above until the "critical" slip surface has been located. The critical slip surface is the one that has the lowest factor of safety and which, therefore, represents the most likely failure mechanism.

Steps *f* and *g* are automated in most slope stability computer programs, but several different starting points and search criteria should be used to ensure that the critical slip surface has been located accurately.

h. Compare the computed factor of safety with experienced-based criteria (see Chapter 3).

Return to any of the items above, and repeat the process through step *h*, until a satisfactory design has been achieved. When the analysis has been completed, the following steps (not part of this manual) complete the design process:

i. The specifications should be written consistent with the design assumptions.

j. The design assumptions should be verified during construction. This may require repeating steps *b, c, d, f, g,* and *h* and modifying the design if conditions are found that do not match the design assumptions.

k. Following construction, the performance of the completed structure should be monitored. Actual piezometric surfaces based on pore water pressure measurements should be compared with those assumed during design (part *d* above) to determine if the embankment meets safe stability standards.

1-7. Unsatisfactory Slope Performance

a. Shear failure. A shear failure involves sliding of a portion of an embankment, or an embankment and its foundation, relative to the adjacent mass. A shear failure is conventionally considered to occur along a discrete surface and is so assumed in stability analyses, although the shear movements may in fact occur across a zone of appreciable thickness. Failure surfaces are frequently approximately circular in shape. Where zoned embankments or thin foundation layers overlying bedrock are involved, or where weak strata exist within a deposit, the failure surface may consist of interconnected arcs and planes.

b. Surface sloughing. A shear failure in which a surficial portion of the embankment moves downslope is termed a surface slough. Surface sloughing is considered a maintenance problem, because it usually does not affect the structural capability of the embankment. However, repair of surficial failures can entail considerable cost. If such failures are not repaired, they can become progressively larger, and may then represent a threat to embankment safety.

c. Excessive deformation. Some cohesive soils require large strains to develop peak shear resistance. As a consequence, these soils may deform excessively when loaded. To avoid excessive deformations, particular attention should be given to the stress-strain response of cohesive embankment and foundation soils during design. When strains larger than 15 percent are required to mobilize peak strengths, deformations in the embankment or foundation may be excessive. It may be necessary in such cases to use the shearing resistance mobilized at 10 or 15 percent strain, rather than peak strengths, or to limit placement water contents to the dry side of optimum to reduce the magnitudes of failure strains. However, if cohesive soils are compacted too dry, and they later become wetter while under load, excessive settlement may occur. Also, compaction of cohesive soils dry of optimum water content may result in brittle stress-strain behavior and cracking of the embankment. Cracks can have adverse effects on stability and seepage. When large strains are required to develop shear strengths, surface movement measurement points and piezometers should be installed to monitor movements and pore water pressures during construction, in case it becomes necessary to modify the cross section or the rate of fill placement.

d. Liquefaction. The phenomenon of soil liquefaction, or significant reduction in soil strength and stiffness as a result of shear-induced increase in pore water pressure, is a major cause of earthquake damage to embankments and slopes. Most instances of liquefaction have been associated with saturated loose sandy or silty soils. Loose gravelly soil deposits are also vulnerable to liquefaction (e.g., Coulter and Migliaccio 1966; Chang 1978; Youd et al. 1984; and Harder 1988). Cohesive soils with more than 20 percent of particles finer than 0.005 mm, or with liquid limit (LL) of 34 or greater, or with the plasticity index (PI) of 14 or greater are generally considered not susceptible to liquefaction. The methodology to evaluate liquefaction susceptibility will be presented in an Engineer Circular, "Dynamic Analysis of Embankment Dams," which is still in draft form.

e. Piping. Erosion and piping can occur when hydraulic gradients at the downstream end of a hydraulic structure are large enough to move soil particles. Analyses to compute hydraulic gradients and procedures to control piping are contained in EM 1110-2-1901.

f. Other types of slope movements. Several types of slope movements, including rockfalls, topples, lateral spreading, flows, and combinations of these, are not controlled by shear strength (Huang 1983). These types of mass movements are not discussed in this manual, but the possibility of their occurrence should not be ignored.